27. Seismic Risks for Embankments

Key Concepts

Case histories indicate there are very few instances where an earthquake has damaged an embankment dam enough to result in the uncontrolled release of reservoir water. Many embankment dams are exposed to earthquake shaking each year, but either the damage caused by the earthquake was not extensive enough, or in the rare cases where damage was extensive, the reservoir was far below the damage and uncontrolled releases did not happen. The failure probability estimation procedures described below are built upon standard analysis techniques used to predict responses of soil to dynamic loading and upon observations from case histories of embankments that have been exposed to earthquakes.

Dynamic loading from an earthquake changes the stress states within an embankment, causing permanent damage if the stress changes cause shear or tensile strength to be exceeded. Loose, saturated, cohesionless soils, when subject to earthquake shaking and initial shearing, can contract as the soil particles are rearranged. Since the water within the pore spaces is virtually incompressible, this results in an increase in pore water pressure. If the pore pressure increase is enough to reduce the effective stress to nearly zero, the soil is said to have liquefied, and the soil experiences a significant reduction in shear strength. Extensive shear strength reduction beneath an embankment slope can trigger a flow slide which, in turn, can result in a very rapid dam failure. In dense, saturated cohesionless soils, large shear displacements may not occur. Instead, the temporary occurrence of excess pore water ratios of 100 percent (or initial liquefaction) is accompanied by the development of limited strains, resulting in progressive and incremental lateral spreading of slopes.

Whether or not the soil of an embankment or its foundation liquefies completely, pore pressure increases can still result in a decrease in shearing resistance. If enough reduction occurs, over a sufficient extent, large deformations can result. A translational failure can occur if the entire foundation beneath an embankment liquefies and the reservoir pushes the embankment downstream far enough to create a gap in the vicinity of an abutment. Overtopping erosion failure can occur if crest deformations exceed the freeboard at the time of the deformations.

If the deformations do not result in an immediate release of the reservoir, the embankment can be cracked or disrupted to the point where internal erosion can occur through the damaged remnant. This failure mechanism can occur with or without liquefaction. There are many ways in which cracking can occur due to seismic shaking, such as differential settlement upon shaking, general disruption of the embankment crest, offset of a foundation fault, or separation at spillway walls. See Chapter 26 on Internal Erosion and Piping Risks for Embankments for other conditions that may make a particular dam more susceptible to transverse cracking and subsequent internal erosion.

Compacted embankments are typically not considered susceptible to liquefaction upon shaking and initial shearing. Dense, cohesionless soils tend to dilate upon shearing, which increases the pore space between soil particles and reduces the pore pressures. Most Reclamation and USACE embankment dams are compacted, so the focus of liquefaction studies tends to be related to loose foundation soils.

However, hydraulic fill embankments may be susceptible to liquefaction or pore pressure increases. Fine-grained soils, while not strictly “liquefiable,” may be susceptible to strength loss during an earthquake. Two aspects of a fine-grained soil's shear strength behavior can require investigation: 1) the anticipated peak magnitude of earthquake-induced shear loading when compared to a soil's undrained shear strength determined from monotonic loading; and 2) sensitivity, which is the potential for a reduction in the undrained shear strength due to the effects of many shearing cycles or very large monotonic strain.

If active faults or faults capable of co-seismic displacement cross an embankment dam foundation, the potential exists for foundation displacement that cracks or disrupts the dam core or water retaining element as well as transition zones or filters. The cracking can initiate concentrated seepage, and the translational movement can create locations where there would be unfiltered exit points for the seepage. Both conditions would increase the likelihood for failure from internal erosion or piping. Shearing of a conduit passing through an embankment dam as a result of fault displacement can result in transmission of high pressure water into the dam, leading to increased gradients and potential for internal erosion. At the time of the 1906 San Francisco Earthquake, Upper and Lower Howell Creek Dams were located on the San Andreas Fault and holding water. Lower Howell Creek Dam which had a conduit through it failed, but Upper Howell Creek with no conduit did not. The presence of the conduit *might* have made the difference.

Seiche waves can be generated by large fault offsets beneath the reservoir, by regional ground tilting that encompasses the entire reservoir, or by mass instability or slope failure along the reservoir rim. “Sloshing” can lead to multiple overtopping waves from these phenomena.

Steps for Risk Evaluation

- Develop detailed site-specific potential failure modes
- Develop event trees to assess the potential failure modes
- Establish loading conditions for earthquake PGA and associated magnitudes, as well the coincident reservoir level
- Evaluate site conditions and develop representative characterization of the embankment and foundation materials
- Perform a screening by evaluating the load combinations and site characteristics to determine if seismic potential failure modes will be significant risk contributors

If the potential failure mode can’t be screened out, then perform the following for each selected earthquake and reservoir load combination

- Estimate the likelihood of liquefaction of any foundation or embankment materials
- Calculate the likelihood of no liquefaction
- Estimate the residual strength of the materials that may liquefy
- Estimate the deformation of the embankment given liquefaction

- Estimate the deformation of the embankment given no liquefaction occurs

For overtopping, assess the estimated deformation, and estimate a probability of overtopping. Different estimates are made for the various reservoir (freeboard) and earthquake combinations represented in the event tree. Complete the event tree nodes following procedures similar to flood overtopping failure modes. See Chapter 16 on Flood Overtopping Failure of Dams.

For cracking, assess the estimated deformation, and determine the likelihood of developing transverse cracks. Estimate the depth and width of the cracks, and complete the event tree similar to the failure mode of internal erosion through cracks. See Chapter 26 on Internal Erosion and Piping Risks in Embankments.

The probability for each node in the event will be determined by team elicitation considering all of the more likely and less likely factors associated with that node. See Chapter 13 on Subjective Probability and Expert Elicitation.

Seismically-Induced Potential Failure Modes

The following are generic descriptions of how a dam might fail due to these potential failure modes. For a specific dam, additional details would be needed in the descriptions, as described in the Chapter 2 on Potential Failure Mode Analysis.

Deformation and Overtopping

Severe earthquake shaking causes loose embankment or foundation materials to contract under cyclic loading, generating excess pore water pressures (i.e., liquefaction occurs). The increase in pore water pressure reduces the soil's shear strength. (This could also occur as a result of loss of strength in a sensitive clay.) Loss of shear strength over an extensive area leads to slope instability and crest settlement. Crest deformation exceeds the freeboard existing at the time of the earthquake. The depth and velocity of water flowing over the crest are sufficient to erode materials covering the downstream slope. Headcutting action carves channels across the crest. The channels widen and deepen. Subsequent human activities are not sufficient to stop the erosion process. The embankment breaches and releases the reservoir. This failure mode can also be initiated without the requirement for liquefaction. If the seismic deformation is great enough for the crest to settle below the reservoir level, overtopping can be initiated. This mostly pertains only to dams that have a small amount of freeboard at the time of the earthquake.

Deformation and Transverse Cracking at the Crest

Severe earthquake shaking causes loose embankment or foundation materials to contract under cyclic loading, generating excess pore water pressures (i.e., liquefaction occurs). The increase in pore water pressure reduces the soil's shear strength. Loss of shear strength over an extensive area leads to slope instability, deformations, and crest settlement. However, crest deformation does not exceed the freeboard existing at the time of the earthquake. Open and continuous transverse cracks form across the crest and through all zones of the dam deep enough to intersect the reservoir. The depth and velocity of water flowing through the open cracks are sufficient to erode the materials along the sides and across the bottom of the cracks. Material from upstream zones is not effective in sealing the cracks (by being transported to a downstream zone or constriction point where a filter would begin to form). Headcutting action carves channels across the crest. The channels widen and deepen. Subsequent human activities are not sufficient to stop the erosion process. The embankment breaches and releases the reservoir. This failure mode can also be initiated without the requirement for liquefaction. If the seismic deformation is great enough for cracking to extend to the below the reservoir level, internal erosion can be initiated. Again, this mostly pertains only to dams that have a small amount of normal freeboard, such as a water supply dam that is kept full most of the time.

Liquefaction and Sliding Opening Gaps

Severe earthquake shaking causes loose embankment or foundation materials to contract under cyclic loading, generating excess pore water pressures (i.e., liquefaction occurs). The increase in pore water pressure reduces the soil's shear strength. (Again, this same outcome could occur if there is sensitive clay in the foundation.) Loss of shear strength occurs in a layer that is continuous upstream to downstream. Reservoir loading exceeds the shearing resistance remaining in the layer, and the entire embankment slides downstream. Downstream deformation opens a gap at the crest deep enough to intersect the reservoir. The depth and velocity of water flowing through the gap are sufficient to erode the materials along the sides and across the bottom of the gap. Material from upstream zones is not effective in sealing the gap (by being transported to a downstream zone or constriction point where a filter would begin to form). Headcutting carves channels across the crest. The channels widen and deepen. Subsequent human activities are not sufficient to stop the erosion process. The embankment breaches and releases the reservoir. It is believed that Sheffield Dam failed by this mechanism in the 1925 Santa Barbara CA earthquake.

Deep Cracking

Severe earthquake shaking causes differential settlement over stiffness discontinuities, at near-vertical embankment-foundation contacts, or at contacts between the embankment and concrete. Continuous transverse cracks of sufficient width form through the core, and concentrate seepage flow through the cracks below the reservoir level occurs. The seepage quantity and velocity are sufficient to erode core material and transport it beyond the downstream shell material. Upstream zones are not effective in sealing the cracks (by a mechanism whereby material from upstream zones would be transported to a downstream zone or constriction point where a filter would begin to form). Subsequent human activities are not sufficient to stop the erosion process. The embankment breaches and releases the reservoir.

Event Tree

Figure 27-1 shows an example event tree for a seismic overtopping failure mode for only one potentially unstable slope (e.g., if a cutoff trench is offset upstream of the centerline, the upstream slope may be more stable than the downstream slope). The first node in Figure 27-1 splits the tree into several branches representing different earthquake loading conditions with selected ranges of peak horizontal acceleration (or other measure of earthquake shaking). The second node splits the tree into branches for different ranges of reservoir elevations at the time of the earthquake with probabilities based on historic or expected future operations (e.g., based on a pool-duration relationship). The third node in Figure 27-1 further separates situations where liquefaction is believed likely or unlikely for a given a peak horizontal acceleration range. If liquefaction occurs, a subsequent node treats conditions where embankment deformations might lead to freeboard loss and failure by overtopping erosion. If liquefaction will not take place, crest deformations are also evaluated and the loss of freeboard is checked for the possibility of overtopping.

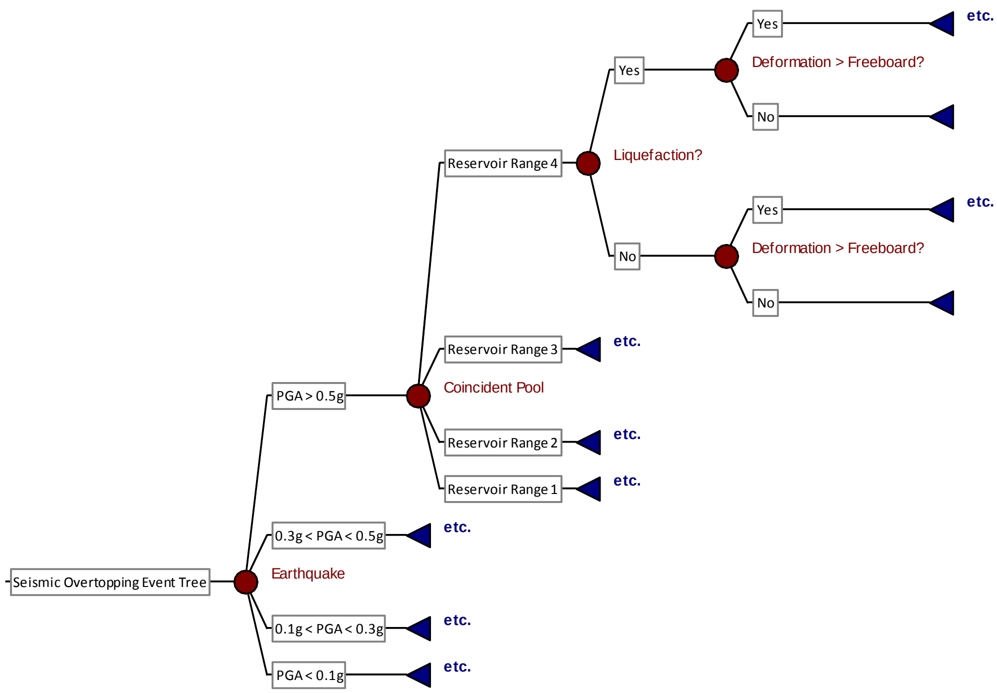


Figure 27-1. Example Seismic Event Tree

The rest of the event tree will be similar to a flood overtopping failure mode and will have additional nodes. For seismic-induced cracking potential failure modes, the deformation node can be replaced with internal erosion nodes, as described in Chapter 26.

Probabilities assigned to events or natural states are multiplied along each branch's pathway, leading to a joint probability for the particular combination of the events or states along that path. Each branch ending in a failure condition contributes to the total failure probability.

Past case history situations where liquefaction has occurred resulted in significantly more extensively damaged embankments. Therefore, failure modes are analyzed in two categories: where liquefaction does and does not take place.

The event tree is rather simple, but complex calculations are made outside the tree and then brought back in. In addition, the events evaluated may be a function of multiple variables such as peak horizontal acceleration, moment magnitude, and coincident reservoir level, or combinations thereof. The steps needed to evaluate the event tree are described in more detail below.

When both upstream and downstream slope stability must be considered, the event tree becomes much more complex. Issues related to estimation of probability of liquefaction (e.g., joint probabilities, independence, and correlation) must be considered for liquefaction and shear strength loss, for the upstream slope and downstream slope separately.

Loading Conditions

Seismic Loading

Larger accelerations and longer durations are generally expected to occur less frequently than small accelerations. Earthquakes can occur randomly within a region of similar seismic activity or be associated with an identified seismogenic fault source. Regional slip rates determine potential earthquake frequency on faults. Statistical models determine earthquake frequency where not associated with a fault. Seismic hazard is typically provided as a return period or an annual exceedance probability for peak horizontal acceleration or in spectral acceleration form at specified periods or period ranges. Acceleration time-history records thought likely to represent specified return period ranges are also used. For the evaluation of liquefaction, the seismic hazard curves need to be deaggregated to determine the magnitude of the earthquakes that have the most contribution to a particular acceleration increment. The USGS has a number of useful tools available on their website for estimating the seismic loads and frequencies. The selection and description of seismic load ranges is covered in the Chapter 6 on Seismic Hazard Analysis and Chapter 11 Event Trees.

Reservoir Loading

Seismic potential failure modes are also a function of the reservoir level at the time of the earthquake. The system response will have to be developed as a function of both the seismic loading and the reservoir loading. The range of reservoir loadings should go from the minimum normal pool to the maximum controllable level, since the duration of storage over an uncontrolled spillway or above designated flood storage with a gated spillway is generally pretty short. USACE will often develop a range of reservoir loadings from the minimum normal pool to the PMF elevation or dam crest for flood control dams using a stage-duration relationship. The frequency associated with the reservoir loading should be based on the stage-duration curve developed for the project. This will give the percent of time the pool is expected to be above a certain elevation.

Site Characterization

Continuity of Liquefiable Materials

The first item to be addressed is the likelihood that a continuous layer or zone of potentially liquefiable material exists within the dam or foundation. This may be explicitly included as a node in the event tree. While simple in concept, estimating the likelihood for continuity requires significant insight. It is typically based on exploratory information and knowledge of the geologic and dam construction processes. For example, the extent of a potentially liquefiable foundation layer is formulated from what is known about the foundation. If the foundation is composed of lacustrine deposits or if the embankment contains hydraulic fill, there would be reason to believe soil properties identified for a layer would in general be laterally continuous. The same may not be true for alluvial stream deposits.

Soil property data, such as Standard Penetration Tests (SPT), Becker-Hammer Penetration Tests (BPT), Shear Wave Velocity Tests (SWV), and Cone Penetrometer

Tests (CPT) can provide insights into the potential for a continuous layer. ***In this regard, the data should be reviewed looking for a continuous low strength layer and not as a population lumped together for statistical analysis.*** The extent of the loose layer can often be constrained to within some limits from this type of data. Then, it becomes a matter of judging the likelihood that the identified layer is continuous enough to lead to a stability problem if it were to liquefy. All field testing should be carefully reviewed to assure that borehole drilling methods or testing methods did not cause significant disturbance that may alter the interpretation of the data. It has been found that improper control of drilling fluid pressures has resulted in borehole heave, and thus subsequent testing indicated false interpretations of low density zones. Whenever very low blow counts are recorded under dams with significant confining pressure, the field drilling and testing methods should be closely scrutinized. Construction photographs, especially of trench excavations, should be used to help assess the continuity and character of the foundation materials.

Typically, continuity parallel to the dam axis of 1 to 2 times the dam height is needed to adversely affect stability without significant three-dimensional effects contributing to stability. If the continuity transverse to the dam axis underlies most of the dam slope, it is probably of sufficient continuity to affect slope stability. Shorter transverse continuity can also affect slope stability depending on the geometry and strength. Slope stability analyses incorporating post-liquefaction shear strengths can be useful in determining how far low-strength materials need to extend beneath a slope before stability becomes an issue.

When there are few of the in situ tests normally used to evaluate liquefaction potential at a site, it has been common to first estimate the likelihood of continuity, and then estimate the likelihood that the zone thought to be continuous can liquefy. When there are many in situ tests, it is common to estimate a range of values for some material property (e.g., SPT blow count or shear strength) related to liquefaction and thought to be "representative" of a zone under the embankment slope that extends laterally 2 to 3 times the height of the embankment. Again, the "representative" value should be judged based on a critical evaluation of the geology and in situ test data, taking care to look for weak zones which have continuity. It is generally best to avoid equating "representative" with a statistical average. Instead, look for an average strength or blow count over a surface drawn through the weakest depths at each drillhole location (supplemented by geologic judgment). A frequent mistake is to take the average of all of the available SPT blow counts in a given geologic unit, regardless of whether the unit appears to have a recognizable low-blow count zone of sufficient extent. Another mistake is to take the average of all of the available data in a unit when borehole spacing is much greater than 2 to 3 times the dam height. In this case, a single low-blow count interval in a single borehole could be significant.

Other Parameters

Along with determining the representative normalized blow counts required to assess liquefaction, many other parameters need to be determined to help evaluate the dam. This includes strengths for non-liquefiable materials, densities, piezometric levels, etc. Additionally, if site response analysis will be done, shear wave velocity measurement may be required. Regional or site-specific fault studies may be appropriate when active faults are present on or near the dam site.

Screening

Screening of seismic potential failure modes can be done by evaluating both the probabilities associated with the load combinations, the characteristics of the dam features, and the embankment and foundation materials. There are factors associated with the loading and dam characteristics that make seismic potential failure modes more likely or less likely.

More likely factors for damaging deformation:

- PHA greater than 0.2g
- Capable faults beneath the embankment
- Hydraulic fill embankments
- Sand embankments
- Loose, saturated alluvial foundations
- Fine-grained soils susceptible to cyclic failure
- Thin impervious cores
- Thin filter zones
- Conduits embedded in embankment
- History of seismic damage
- Earth embankment-concrete section interface

Less likely factors for damaging deformation

- PHA less than 0.2g
- No capable faults beneath embankment
- Well-built, rolled/compacted embankments (i.e., RC > 95 percent or D_r > 75 percent)
- Non-liquefiable embankment and foundation materials (i.e., embankment founded on rock, dense foundation soils with $(N_1)_{60}$ > 30 bpf, or foundation materials are non-sensitive clays)
- Unsaturated soils
- Embankment slopes flatter than 3H:1V
- Large core and filter zones
- Rock fill shells
- Static factor of safety against slope instability greater than 1.5
- Freeboard greater than 3 to 5 percent of the embankment height and low seismicity
- No embedded critical features that would be harmed during small embankment movements

The risk assessment team should also assess the combined probabilities of the seismic and reservoir loads early in the process. Often for flood control dams (or dams with flatter slopes and large normal freeboard), the seismic potential failure modes can be screened out just on the basis that the loading required to make the failure mode credible is so remote that it will not drive the project risk.

A good knowledge of case histories related to dam performance during earthquakes is essential to help guide the judgment of the risk assessment team.

Case Histories

Relatively few dams have actually failed as a result of liquefaction, internal erosion through seismically-induced cracks, or other seismic-related failure modes. However, a few case histories provide relevant insights.

Lower San Fernando Dam: 1971

The upstream slope of Lower San Fernando Dam failed during the 1971 San Fernando Earthquake (Seed et al., 1975). Intact blocks of embankment material moved tens of feet on liquefied hydraulic fill shell material. There was evidence to suggest the slope failure took place after the shaking had stopped. Fortunately, a remnant of the dam remained above the reservoir water level at the time, and the dam did not breach.

Sheffield Dam: 1925

Sheffield Dam failed during the Santa Barbara earthquake of 1925. Although there were no witnesses to the breach, it was believed that the sandy foundation soils which extended under the entire dam liquefied and that a 300-foot long section of the dam slid downstream, perhaps as much as 100 feet (Seed et al., 1969). The dam was located quite close to the city of Santa Barbara, and a wall of water rushed through town, carrying trees, automobiles, and houses with it. A muddy, debris-strewn aftermath was left behind. Flood waters up to 2 feet deep were experienced in the lower part of town before they gradually drained away into the sea. No fatalities were reported.

Austrian Dam: 1989

Austrian Dam was severely cracked and damaged by the 1989 Loma Prieta Earthquake (Forster and MacDonald, 1998), with peak ground accelerations estimated at 0.5g to 0.6g from the nearby magnitude 7 event. Longitudinal cracks that were 14 feet deep (based on trenching) formed just below the dam crest on the upstream and downstream slopes. Transverse cracks formed at both abutments, 1 to 9 inches wide, and the embankment separated from the concrete spillway wall, opening a gap of about 10 inches. Fortunately, the reservoir was low at the time of the earthquake, and no subsequent internal erosion ensued.

San Fernando Power Plant Tailrace Dam: 1994

A small embankment dam forming the tailrace for the San Fernando power plant was shaken by large ground motions during the 1994 Northridge earthquake. The earthquake occurred early in the day, and the tailrace dam was intact when power plant personnel left for the day. The next morning, the dam had failed (Davis, 1997). The tailrace concrete lining had buckled in several locations. It was suspected that a layer of loose sand beneath the dam, identified by CPT data, liquefied, and piped through the gaps in the concrete lining undetected, slowly throughout the day.

Cracking in Dams Exposed to Loma Prieta Earthquake: 1989

Harder (1991) lists the damage that occurred to 35 dams exposed to the Loma Prieta Earthquake. The completion date, maximum dam height, distance to the epicenter, and estimated peak ground accelerations are included along with the damage descriptions. The Loma Prieta Earthquake was a magnitude 7.0 earthquake with approximately 7 to 10 seconds of strong shaking. Dams exposed to less than 0.2g did not experience damage. Dams exposed to peak ground accelerations between 0.2g and 0.35g either experienced no damage or developed longitudinal cracks. Transverse cracking was only noted in dams exposed to greater than 0.35g, although 7 of 19 dams exposed to this level of shaking experienced no damage, 7 of 19 dams experienced either minor or longitudinal cracking, and only 5 of 19 dams experienced transverse cracking.

Likelihood of Liquefaction

Estimating the likelihood of liquefaction for any given zone or layer depends on several factors and requires computations outside of the event tree. It is not the intent of this section to provide a detailed discussion of liquefaction evaluation. See the embankment dam draft seismic design standard (Reclamation, 2001), USACE's Draft EC 1110-2-6001 Seismic Analysis of Embankment Dams, Seed et al. (2003), Idriss and Boulanger (2008, 2010), Bray and Sancio (2006), and Boulanger and Idriss (2004) for more information.

Several analyses need to be conducted before the risk assessment occurs. The a_{max} will need to be determined from site response analysis or approximations. The cyclic stress ratio will need to be calculated for each particular load level and at key locations beneath the dam. In addition, raw blow count data will need to be normalized and corrected for fines content. If CPT or shear wave velocity data is to be used, that information must be reduced and normalized. See Idriss and Boulanger (2008, 2010) or Seed et al. (2003) for a discussion of these methods.

Bray and Sancio (2006) report on how soils of differing plasticity index demonstrate liquefaction susceptibility. Boulanger and Idriss (2004, 2008) provide additional guidance on liquefaction and post-liquefaction behavior of fine-grained soils.

Probabilistic liquefaction models are all based on statistical regressions using corrected SPT $(N_1)_{60}$ blow count, fines content (FC) or percent passing the No. 200 sieve, and cyclic stress ratio (CSR) as the basic input parameters: Liao et al. (1988), Youd et al. (2002), Cetin et al. (2000 and 2004), and Seed et al. (2003). Idriss and Boulanger (2010) is the most recent relationship developed from a thorough, updated re-examination of the case history database and database of cyclic test results for frozen sand samples. It is considered to be a technical supplement to Idriss and Boulanger (2008).

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{max}}{g}r_d\frac{1}{MSF}\frac{1}{K_\sigma}$$

$$P_L\left((N_1)_{60cs}, CSR_{M=7.5,\sigma'_v=1atm}\right) = \left[\frac{\frac{(N_1)_{60cs}}{14.1}+\left(\frac{(N_1)_{60cs}}{126}\right)^2-\left(\frac{(N_1)_{60cs}}{23.6}\right)^3+\left(\frac{(N_1)_{60cs}}{25.4}\right)^4-2.67\text{-}\ln\left(CSR_{M=7.5,\sigma'_v=1atm}\right)}{0.13}\right]$$

A distribution of $(N_1)_{60}$ can be developed to represent the potentially liquefiable layer or zone of interest. It may be appropriate to examine more than one distribution, depending on the available information. A similar approach can be used to develop a probability distribution for fines content. A spreadsheet can then be used to calculate the probability of liquefaction using the $(N_1)_{60}$ and FC distributions.

Once the probability of liquefaction is determined, then the probability of no liquefaction is calculated as one minus the probability of liquefaction.

Residual Shear Strength of Liquefied Soil

An estimate of the residual shear strength of the liquefied materials is needed to estimate deformation. Several empirical relationships have been published that correlate residual undrained shear strength of liquefied material with standard penetration test resistance. The most common relationships used in practice include Seed and Harder (1990) as shown in Figure 27-2 and Olson and Stark (2002) as shown in Figure 27-3. The Seed and Harder (1990) relationship requires an equivalent clean sand SPT-corrected blow count in which blow counts are added to the $(N_1)_{60}$ value according to interpolations from Table 27-1. The Olson and Stark (2002) relationship does not include a fines content correction.

Within the very limited case history database, most instances of flow liquefaction have occurred at fairly shallow depths (i.e., low effective overburden pressure), and none had an $(N_1)_{60\text{-}cs}$ value above 14. It is likely that the lack of embankment flow liquefaction cases for the medium -to high-blow count materials is related to the fact that high blow count materials are dilative and the medium blow count materials which may be initially contractive become dilative with strain. The Seed and Harder (1990) relationship does not allow for any beneficial effects from higher effective overburden stress, common beneath large embankment dams. In an attempt to account for this effect, Seed et al. (2003) recommended using a weighted average of the residual undrained shear strength values from Seed and Harder (1990) at 80 percent and Olson and Stark (2002) at 20 percent. Idriss and Boulanger (2008) used blow counts and strength estimates from both Seed and Harder (1990) and Olson and Stark (2002) to develop two relationships for extrapolation beyond the limits of available data as shown in Figure 27-4: one for use in situations with potential for upward migration of voids to create a fluidized zone just below a less pervious layer and one for situations without that potential. The two curves are essentially the same within the limits of available data.

Gillette (2010) reviewed the various relationships in an attempt to determine the most appropriate correlation between overburden and blow count. The strength-ratio approach appears to work better at higher effective overburden stresses (i.e., exceeding 1,000 to 1,400 psf) than it does at lower ones. For medium-density soils, those dense enough to dilate at larger strains after initial liquefaction, the strength ratio is thought to be the most realistic model, as the shearing resistance increases with larger strain and becomes a large fraction of the drained strength. However, care must be taken in selection of undrained residual shear strength from such correlations given the very limited data at higher effective overburden stresses and within alluvium as opposed to other materials that have liquefied.

Table 27-1. Blow Count Corrections to Obtain Clean Sand Equivalent (Seed 1987)

Fines Content (percent)	Blow Counts (bpf) added to $(N_1)_{60}$
10	1
25	2
50	4
75	5

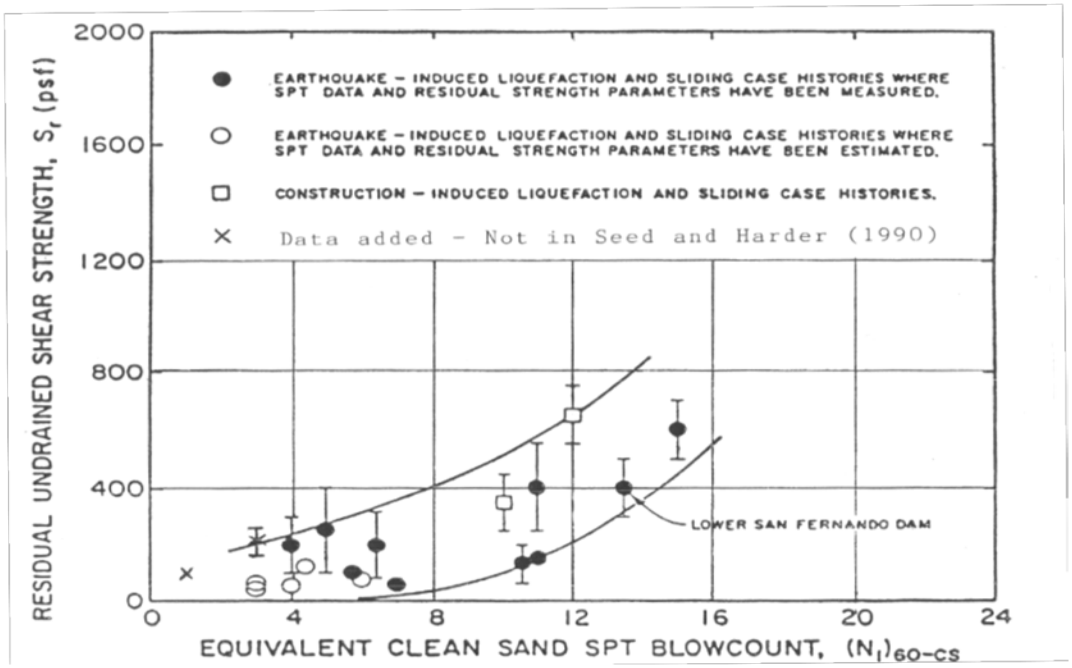


Figure 27-2. Residual Undrained Shear Strength
(adapted from Seed and Harder, 1990)

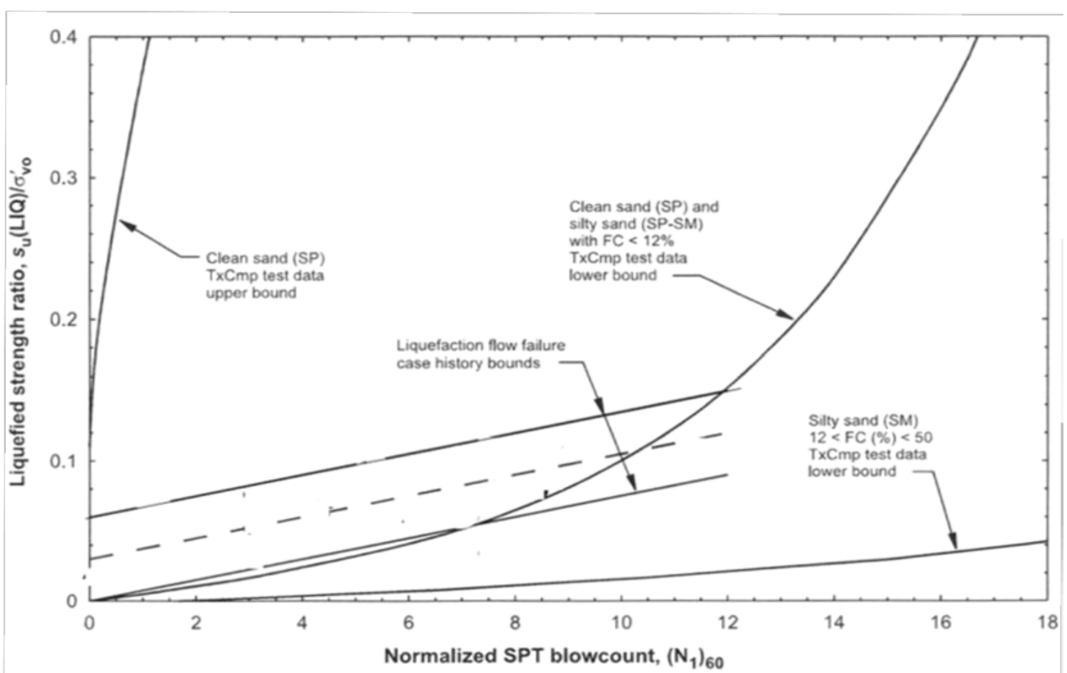


Figure 27-3. Normalized Residual Shear Strength Ratio of Liquefied Soils
(adapted from Olson and Stark, 2002)

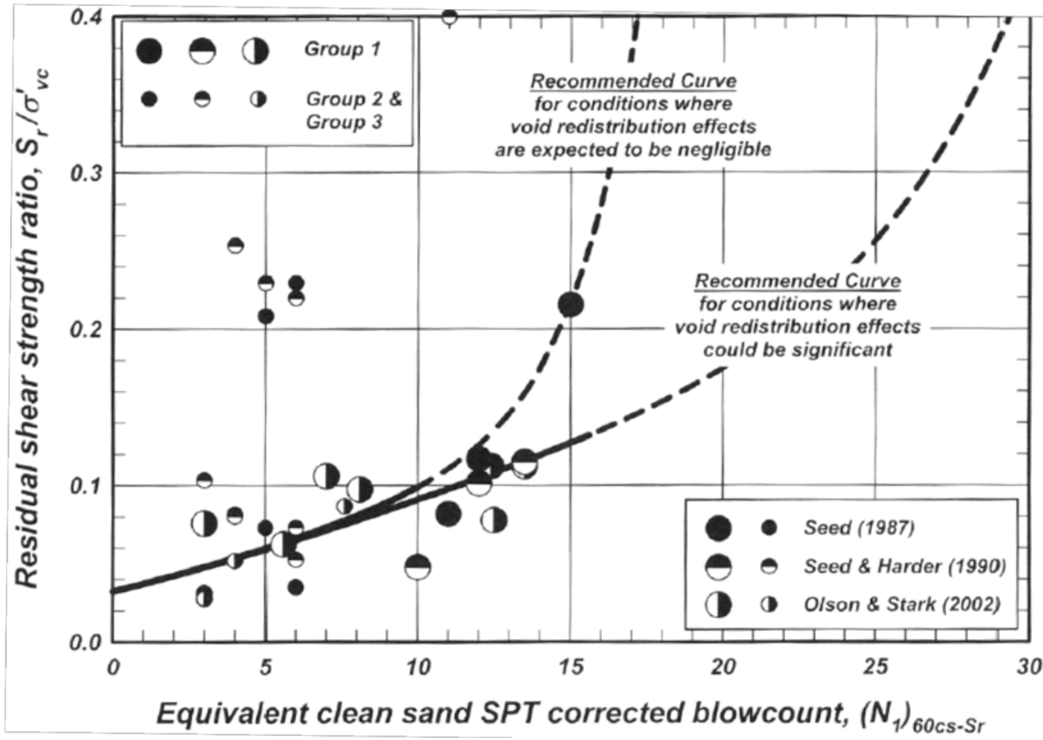


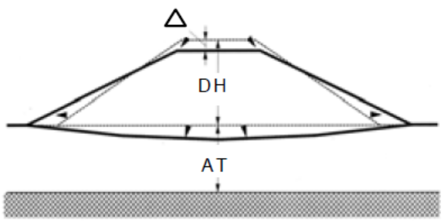
Figure 27-4. Normalized Residual Shear Strength Ratio of Liquefied Soils (Idriss and Boulanger, 2007)

Embankment Deformation

There are numerous methods used to estimate deformations of embankments in response to seismic loading. Unfortunately none of the methods, (including rigorous models) have been proven to accurately predict actual deformation shape and magnitudes. The risk assessor must be familiar with the assumptions and limitations of the methods used to estimate the embankment deformation and apply significant judgment when assessing the probability associated with deformation-related potential failure modes. Simplified methods should be used first. If an evaluation using one of the simplified methods results in an estimated annual probability of failure or annualized incremental life loss that exceeds risk guidelines, more refined studies are probably justified. This requires detailed FLAC analyses to estimate the loss of freeboard due to various seismic loads. Typically, enough FLAC analyses are run to develop curves (high, median, and low) for freeboard loss as a function of residual undrained shear strength of the liquefied layers or zones. Team judgment incorporating model uncertainty is also included in the development of the curves.

Empirical Deformation (No Liquefaction Occurs)

If liquefaction does not occur, movements that occur within the dam body without distinct signs of shearing displacement can lead to deformation that exceeds the available freeboard. Swaisgood (1998, 2003) examined case histories of seismic-induced settlement and mass deformation where the earthquake shaking causes embankments to settle downward and sidewards, towards the deepest center portion of the valley, and then spread upstream and downstream away from the dam axis. In the Swaisgood (2003) empirical methodology, the crest settlement is expressed as a percentage of the total embankment height and foundation thickness, as shown in Figure 27-5a. The crest settlement (given that no liquefaction occurs) is given as a function of peak ground acceleration (PGA) and surface wave magnitude (M_s) as shown in Figure 27-5b. ***The incident database does not contain any cases with PGA greater than 0.7g and normalized settlements greater than 5 percent. However, some incidents involving liquefaction were included in the database: Hebgen Dam (1959), Upper San Fernando Dam (1971 and 1994), and Masiway Dam (1990). Austrian Dam (1989) did not experience liquefaction but had other issues like poor compaction and an existing slide left in place in one abutment. If these cases are excluded, the incident database does not contain any cases with normalized settlements greater than 1 percent.***



$$\% \text{ STTLMT} = \frac{\Delta}{DH + AT} \times 100$$

Figure 27-5a. Crest Settlement (Swaisgood, 2003)

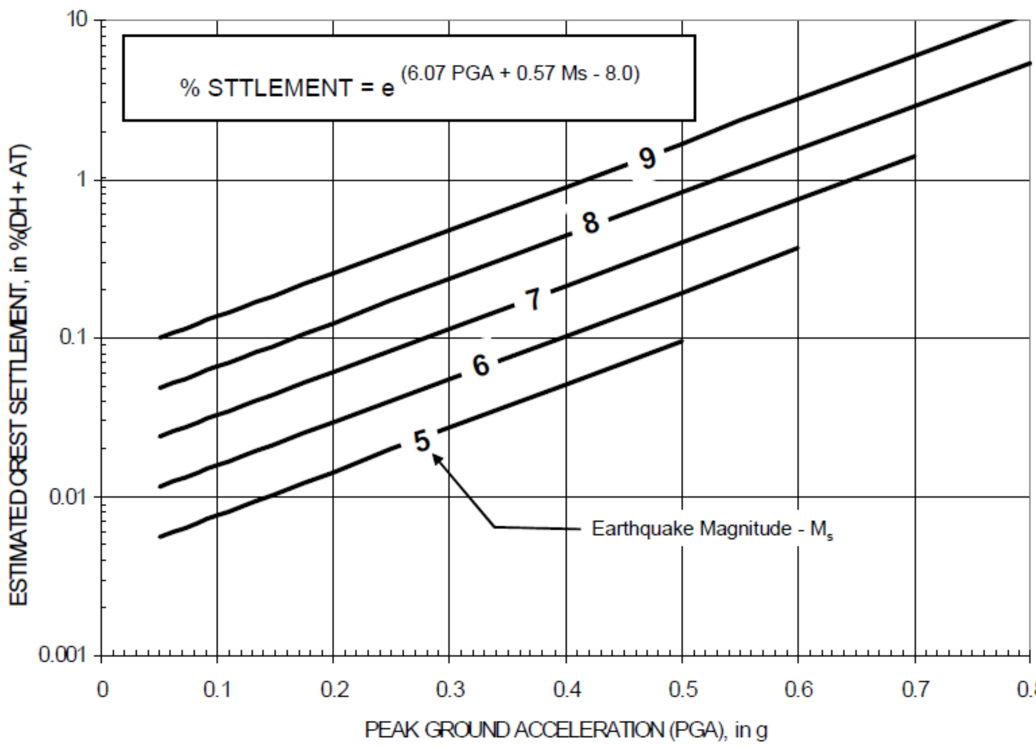


Figure 27-5b. Estimated Crest Settlement (Swaisgood, 2003)

Simplified Dynamic Slide Mass Deformation

Newmark (1965) developed a method for estimating the displacement of a slide mass due to dynamic shaking based on the assumption that permanent displacement occurs when the dynamic stress exceeds the shear resistance along the sliding mass. This method has been modified and updated by others including: Makdisi and Seed (1978), Watson-Lamprey and Abrahamson (2006), and Bray and Travasarou (2007). Limitations include:

- Deformation is only assumed to occur only along the sliding surface and not as shear strain throughout the embankment.
- Deformation assumed to only occur during the shaking
- Only valid for non-liquefied embankment and foundation materials

Post-Earthquake Stability Analysis

Limit equilibrium slope stability modeling can be used to assess the likelihood that the embankment will have significant deformation and discern a very approximate value for maximum crest deformation when the embankment slope has a factor of safety less than 1. The factor of safety of a dam slope should be determined given the potentially liquefiable zones are at their residual shear strength. The failure surfaces evaluated should only be for significant slide planes that would influence the performance of the dam and lead to potential breach. If the factor of safety is determined to be in the 1.2 to 1.3 range, it is likely that the embankment will not develop significant displacement. If the factor of safety is less than or equal to1.1, then it can be assumed that the slide plane deforms significantly, and the reservoir is held back only by the remnant embankment behind the sliding mass. Essentially, this remnant of relatively undisturbed embankment material would provide the highest remaining barrier to uncontrolled reservoir release. The peak of the undisturbed remnant could be used to assess the likelihood of overtopping. Figure 27-6a shows a series of circular and wedge-shaped failure surfaces analyzed using a limit equilibrium method. Figure 27-6b shows the same cross section modeled using FLAC. The deformation arrows are absent in Figure 27-6b on the downstream slope at a point where the Figure 27-6a shows a failure surface that has a factor of safety Factor of 1.12. The FLAC analysis shows highly deformed material remaining above the elevation of the peak of the undeformed section. An estimate of the remnant crest can be made by assuming that all of the slide mass moved below the scarp intersection with the embankment. The likelihood of attaining a safety factor along such a surface less than this can be estimated using reliability analysis (see Chapter 12 on Probabilistic Stability Analysis (Reliability Analysis)) using a software program like SLOPE/W which can be run in a probabilistic mode. In general, the process is very similar to performing a conventional stability analysis, but instead of defining the input parameters as discrete values, they are characterized as random variables with a probability distribution. A Monte Carlo simulation is used to determine the probability of obtaining a factor of safety less than 1.0.

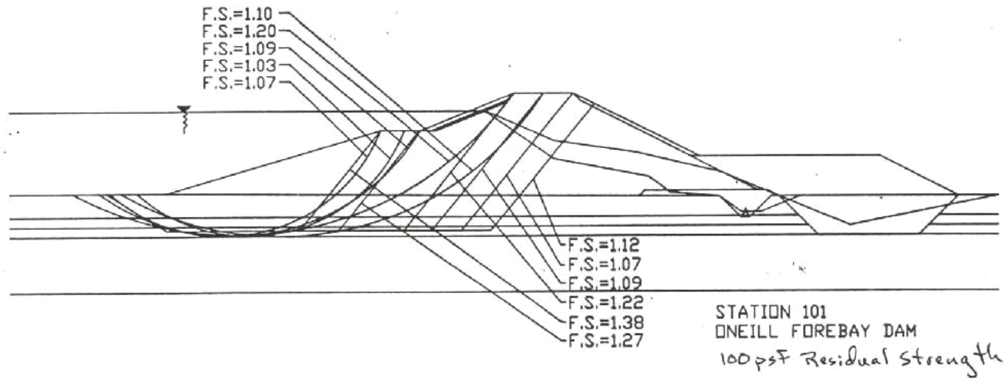


Figure 27-6a.

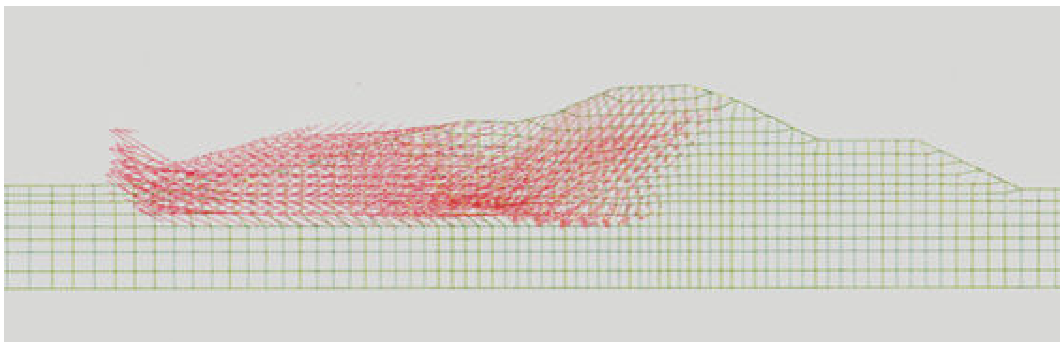


Figure 27-6b.

Simplified Post-Earthquake Deformation

To provide a tool for a quick, screening-level assessment of post-earthquake deformation, USACE and FMSM (2007) performed a parametric study to develop simplified equations. It was assumed that the post-earthquake static deformation is the primary contributor and deformation during shaking was not evaluated. The simplified toolbox was based on a parametric analysis of over 20,000 cases using FLAC. Six variables were considered: height of embankment (H_{emb}), thickness of liquefied foundation soil (H_{liq}), side slopes (m_{side}), normalized depth of pool (h_{pool}) , shear strength ratio of embankment soil (r_{emb}), and shear strength ratio of liquefied foundation soil (r_{liq}). Of the 20,000 cases evaluated only 8,612 (43 percent) resulted in a valid converged solution. Solutions were not obtained for cases where the embankment was unstable before liquefaction, and convergence was not obtained for cases with severe localized distortion. ***Only the valid converged cases were used to develop a regression equation to estimate deformation.*** The regression equation for crest deformation determined by FMSM represents the difference in the elevation between the initial embankment crest and the highest valid grid point on the surface of the deformed embankment computed in the FLAC model. For screening-level purposes, the crest deformation of an embankment given liquefaction occurs is estimated using the following expression from FMSM (2007):

$$\log_{10}(\Delta) = -6.399 \cdot H_{emb}^{-1} - 0.6023 \cdot \log_{10}(r_{emb}) + 1.581 \cdot \log_{10}(m_{side})$$
$$- 4.689 \cdot h_{pool} H_{emb}^{-1} + 0.9123 h_{pool}^{3} - 6.256 \cdot H_{liq}^{-1} - 8.428 \cdot r_{liq} + 2.620$$

The reported R^2 value for the regression equation is 0.803. Valid ranges are specified by FMSM (2007) for the input parameters and irregular or asymmetrical embankment cross sections. The variables are defined in Figure 27-7. Only basic geometries can be evaluated with this method. To use this tool with multiple layers or complex geometries, an equivalent simplified cross section must be developed, as described in FMSM (2007). The loss of freeboard needs to be compared to the reservoir elevation at the time of the earthquake.

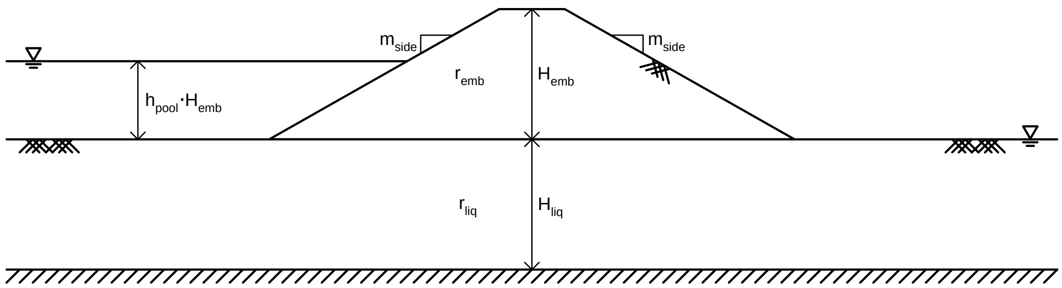


Figure 27-7. "Generalized" Cross Section (FMSM, 2007)

Numerical Post-Earthquake Deformation

If the risk assessment team has an experienced modeler available, then performing a post-earthquake static deformation analysis using a computer program like FLAC can be very valuable. The materials that are potentially liquefiable are modeled at their residual undrained shear strengths. Only gravity loading is applied, and the deformed shape and displacement magnitudes are determined. This analysis neglects the potential deformation that could occur during the shaking. Many observations of embankment instability from seismic loadings have indicated that most of the deformation actually occurs after the shaking stops. This analysis is much less complicated, when compared to the issues of modeling the deformation during dynamic shaking, and are generally considered more reliable.

Numerical Dynamic and Post-Earthquake Deformation

The computer program FLAC can also be used to analyze seismically-induced deformation. FLAC is a two-dimensional explicit finite-difference program. This program can be used to simulate the behavior of structures built of soil, rock, or other materials that may undergo plastic flow when their yield limits are reached. Materials are represented by zones and regions that may be shaped by the user to conform to the physical structure being modeled. Each zone is assumed to behave according to a prescribed linear or nonlinear stress/strain law in response to applied forces or boundary constraints. The represented material can yield and flow, and the grid can deform and move with the material being represented. However, caution and experience are needed when using such sophisticated nonlinear computer programs to ensure the results are reasonable. The models should be thoroughly tested, validated, and verified to ensure reasonable performance. Parametric evaluations should be performed to make sure the model is producing results that intuitively seem to match the expected behavior. The results of this testing should be documented so that those reviewing the results of the analyses will have as much confidence as possible in the results. Model uncertainty can be included in the probability estimates rather than strictly relying on the output numbers (e.g., to account for three-dimensional effects if two-dimensional models were used).

Deformation can result from slope instability under gravity loading alone. If an earthquake can trigger liquefaction, pore water pressure increases reduce shear strength, and the slope might become unstable. After liquefaction triggering, a slope can continue to deform even though the earthquake shaking has ceased if the static factor of safety is less than 1. Should liquefaction initiate early in the earthquake, continued shaking provides inertial forces that add to deformation. Modeling experience using FLAC has shown that when the static factor of safety is less than 1, the dynamic deformation portion is typically a small fraction of the total deformation. Intuitively, the dynamic component will be more significant for earthquake acceleration records of long duration, particularly when the earthquake provides strong accelerations with long periods (as indicated by high spectral acceleration for long periods, such as 1 second).

Resource constraints usually dictate that FLAC results are generated for a limited number of loadings and assumed initial conditions. For the example in Figures 27-8a and 27-8b below, a foundation layer beneath an embankment slope was assigned residual shear strength values of 50, 100, and 200 psf. Gravity loading alone produced the deformation values labeled “Static.” A relatively strong earthquake was responsible for the additional deformation labeled “Dynamic.” Connecting the six model point-estimates with lines, as shown in Figure 27-8a, is reasonable. One could easily analyze the model with additional parameter assumptions to fill in the spaces between previous runs. Likewise, extrapolating the lines to the right, as shown in Figure 27-8b, is appropriate, and we would expect verification with additional analysis for higher shear strength values. Extrapolation to the left as shown in Figure 27-8b is much more problematic, especially if the post-earthquake static factor of safety approaches or falls below 1.0. In that case, there is a transition between two general types of behavior, dynamic deformation occurring only during strong shaking, and gravity-driven slope instability. Limit-equilibrium slope stability (SLOPE/W or similar program) may be necessary before extrapolating.

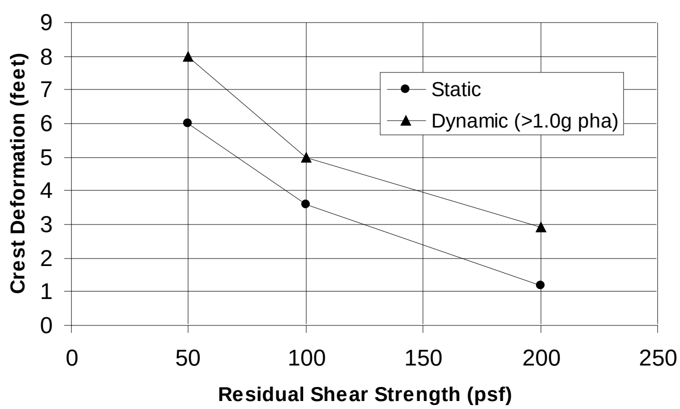


Figure 27-8a.

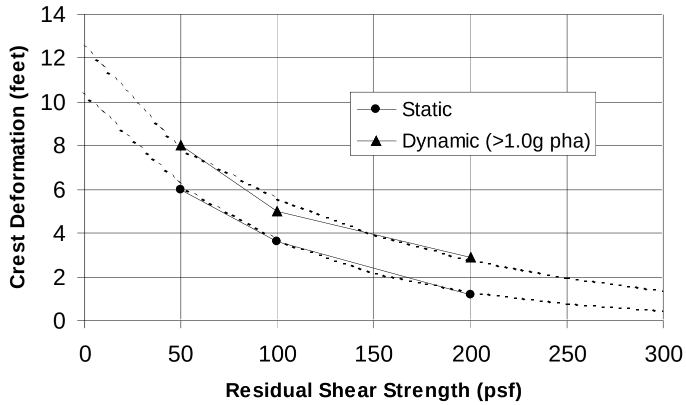


Figure 27-8b.

Overtopping (Deformations exceeding Freeboard)

The probability of overtopping is typically estimated by developing curves of expected deformation. For example, the risk assessment team may estimate the range of absolute minimum crest settlement, reasonable minimum settlement, best estimate or median settlement, reasonable maximum settlement, and absolute maximum crest settlement. These values then form a probability distribution of crest settlement for the strength loss value assumed to result from liquefaction or cyclic failure. If the reservoir remains relatively constant, the deformation curves represent the likelihood of losing a particular amount of freeboard, which can be compared to the freeboard prior to the earthquake in order to assess the likelihood of a breach. If the reservoir fluctuates considerably, the operations cycles are reviewed to get a feel for the percent of time the reservoir is above a threshold level for seismic failure modes to initiate using a pool-duration relationship can be used to represent the coincident pool at the time of the earthquake. The seismic hazard curve performs the annualization, and the pool-duration curve provides the fraction of a given year that the pool is equaled or exceeded.

In some cases, branches for the continuity of liquefiable materials, strength loss, and deformation exceeding freeboard are combined by considering the probability of a given strength scenario, and the resulting deformations given each strength scenario. Specifically, the first two probabilities (probability of continuity and probability of strength loss) are instead phrased as the probability that a given strength will result from a given increment of earthquake loading. This is particularly useful when the risk assessment team has developed deformation models for several different strength scenarios. The strengths assigned in these scenarios are meant to model a likely range of values and include reasonable upper and lower bounds. For example, if Newmark and/or FLAC analysis have been performed for three different strength assumptions, the risk assessment team estimates the likelihood of each of the three strength assumptions, with the sum of the three probabilities equal to 1.0. Expected deformation curves for each of the three strength scenarios can then be developed as described above. This approach is useful in allowing teams to reflect the (sometimes considerable) uncertainty in estimating the strength loss (and corresponding deformations) that will result from earthquake shaking.

Internal Erosion through Cracks

If the embankment and foundation do not liquefy or if the freeboard is not completely lost through seismic deformations, the dam will not fail due to overtopping (or rapid erosion of the severely damaged dam crest), but there is still the potential for a slower internal erosion through cracks in the embankment, typically in the crest and upper portions of the dam. Fell et al. (2008) include considerations for internal erosion through seismically-induced cracks based in part on observed damage to embankment dams following large earthquakes. The primary goal is to determine how deep the embankment is likely to crack and how open the cracks are likely to be below the reservoir surface. Once this is determined, the likelihood of internal erosion is assessed in a similar fashion as for flood loading.

The methodology of Fell et al. (2008), described below, can be used as a tool to assess the likelihood of a crack due to earthquake shaking. The first step in the procedure is to determine the damage class from Figure 27-9. This typically requires deaggregation of the seismic hazard to determine the magnitudes of the earthquakes that contribute most to the hazard at various peak horizontal ground accelerations. If liquefaction occurs, Damage Class 3 or 4 can be assumed, depending on the severity of the estimated liquefaction. Fell et al. (2008) suggest assuming Damage Class 4 if flow liquefaction occurs and Damage Class 3 if liquefaction occurs but it is not flow liquefaction. A Damage Class is determined for each earthquake load partition. It is often desirable to develop a separate event tree to evaluate internal erosion through cracks (as opposed to tacking it on to the end of the liquefaction tree at the non-breach nodes). If a separate tree is developed, care must be taken in combining these risks with liquefaction overtopping risks (and other seismic risks), as discussed in Chapter 35 on Combining and Portraying Risks (common-cause adjustment) so as to not assign a combined conditional failure probability that is too high for a given load range. Given the Damage Class, determine the likely settlement as a percentage of dam height from Table 27-2. Cracking begins at the new elevation of the crest after seismically-induced settlement and extends downward from there.

The probability of transverse cracking and the likely crack width at the crest can be estimated from Table 27-3, which shows the range of values suggested in Fell et al. (2008). Given the crack opening at the crest, the crack width at various depths below the crest and the probability of initiation can be estimated using the procedure described in the Chapter 26 on Internal Erosion and Piping Risks for Embankments.

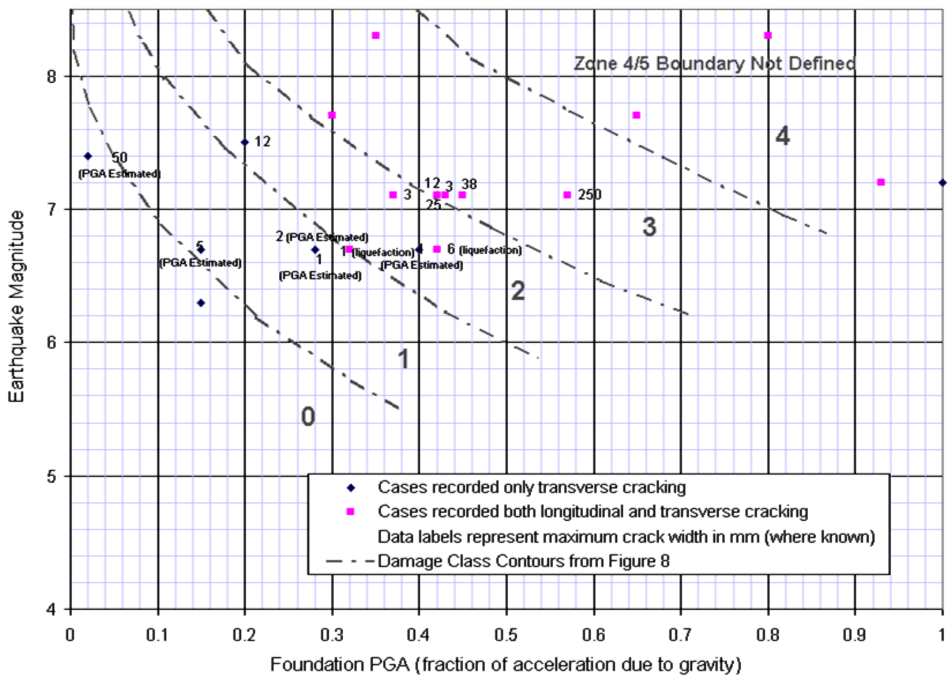


Figure 27-9a. Incidence of transverse cracking versus seismic intensity and damage class contours for earthfill dams (Pells and Fell, 2003)

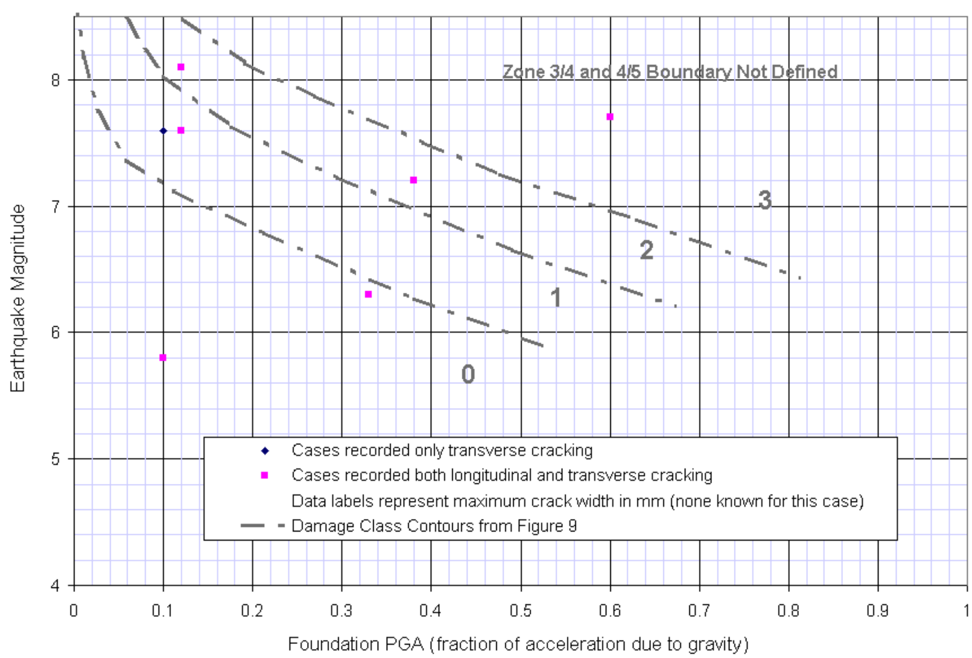


Figure 27-9b. Incidence of transverse cracking versus seismic intensity and damage class contours for earthfill and rockfill dams (Pells and Fell, 2003)

Table 27-2 Damage Classification System (adapted from Pells and Fell, 2002, 2003)

Damage Class		Maximum Longitudinal Crack Width [1] (mm)	Maximum Relative Crest Settlement [2] (percent)
Number	Description		
0	No or Slight	< 10	< 0.03
1	Minor	10 to 30	0.03 to 0.2
2	Moderate	30 to 80	0.2 to 0.5
3	Major	80 to 150	0.5 to 1.5
4	Severe	150 to 500	1.5 to 5
5	Collapse	> 500	> 5
Notes: (1) Maximum likely crack width is taken as the maximum width of any longitudinal crack. (2) Maximum relative crest settlement is expressed as a percentage of the structural height.			

Table 27-3 Probability of Transverse Cracking (adapted from Fell et al. 2008)

Damage Class		Probability of Transverse Cracking	Maximum Likely Crack Width at the Crest (mm)
Number	Description		
0	0.001 to 0.01	5 to 20	< 0.03
1	0.01 to 0.05	20 to 50	0.03 to 0.2
2	0.05 to 0.10	50 to 75	0.2 to 0.5
3	0.2 to 0.25	100 to 125	0.5 to 1.5
4	0.5 to 0.6	150 to 175	1.5 to 5

Foundation or Reservoir Fault Displacement

Where an active fault or fault capable of coseismic displacement exists in the foundation of a dam, offset along the fault can cause cracking of the embankment and/or conduits passing through the dam. Since each dam and geometry is unique, a site-specific event tree needs to be developed to evaluate this on a case-by-case basis. The loading in this case involves fault offsets of various magnitude ranges and their associated probability. Input from Quaternary geologists specializing in fault and seismic source characterization is typically needed to develop this input. An event tree is developed to describe the specific potential failure mode being evaluated. Nodes on the tree would include all of the component events required to cause failure of the dam by this mechanism, and their likelihood. This would include, for example, the likelihood of a through-going crack given some amount of fault offset, or of the embankment filter being disrupted, given that the through-going crack has formed.

Bray et al. (2004) provides an analytical method for preliminary estimates of the height of the shear rupture zone in saturated cohesive soils overlying a bedrock fault displacement based on field observations and physical model experiments. The results indicated that propagation of the shear rupture zone in the overlying soil at a specific bedrock fault displacement depends primarily on the clay's axial failure strain, as shown in Figure 27-10, where the height of the shear rupture zone in the clay overlying the bedrock fault has been normalized with the magnitude of the vertical base displacement. The rupture zone propagates farther in saturated clayey materials that exhibit brittle stress-strain behavior (i.e., low values of failure strain). The orientation of the shear rupture zone through the soil depended largely on the orientation of the underlying bedrock fault plane. The final shear rupture zone in the clay tended to follow the projection of the bedrock fault plane, although there was a tendency for the rupture zone to increase in dip as the rupture zone approached the ground surface and to widen slightly.

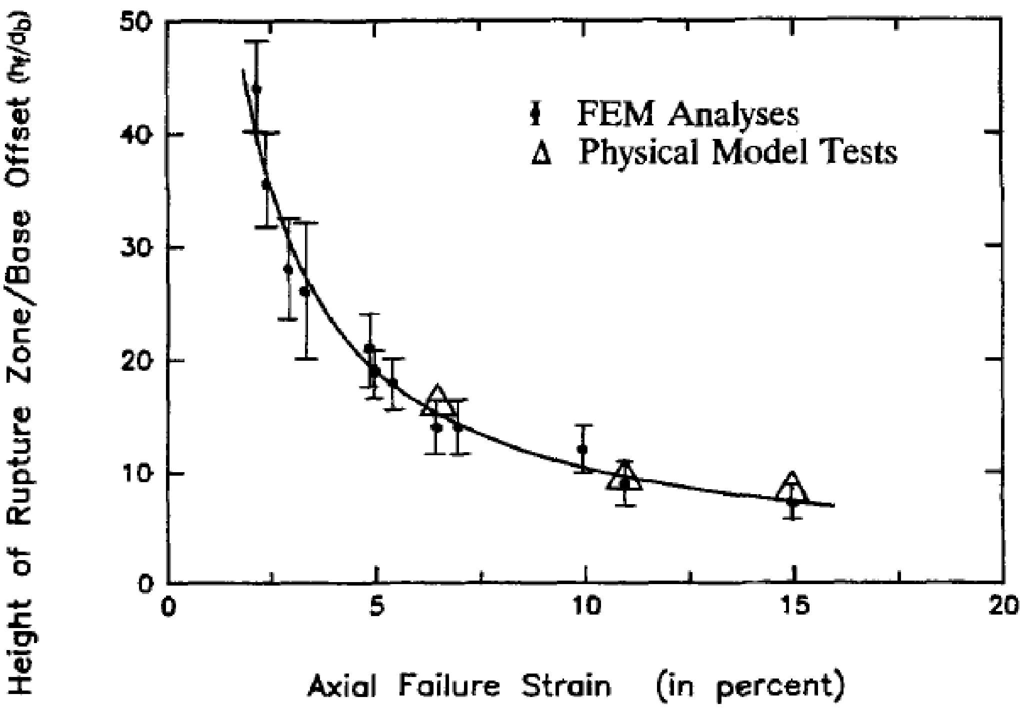


Figure 27-10. Estimated Normalized Height of Shear Rupture Zone as a Function of Clay's Axial Failure Strain (Bray et al., 2004)

An active fault may pass through the reservoir. Fault offset within the reservoir could create a seiche wave capable of overtopping and eroding the dam. Again, it is necessary to develop an event tree, establish return periods for various levels of fault offset, assess the potential for an overtopping wave to develop, and evaluate the likelihood of short duration overtopping to lead to an erosional breach. An initial estimate of wave height equal to the vertical fault offset is probably reasonably conservative in most cases. The reader is referred to Wilson (1972) and Hammack (1973) for additional discussion on modeling seiche waves. However, overtopping failure of a dam due to seiche waves is a relatively improbable failure mode which is only considered when seismotectonic specialists indicate a high likelihood for development of a seiche wave.

Accounting for Uncertainty

Uncertainty is accounted for in the risk calculations by assigning probability distribution functions for important variables in the risk analysis, such as the representative SPT blow count, the amount of deformation that would occur with a given loading, or the probability of some event, such as the embankment filter being disrupted. Spreadsheet cells containing input values are described in terms of a distribution rather than a single value. Then, a Monte Carlo simulation is performed (typically with 10,000 iterations) to develop a probability distribution for the annual failure probability and annualized loss of life. In some cases, the Monte Carlo model may require calculations and sampling of parameters outside of the event tree. For example, the probability of liquefaction, crest deformation (settlement), and the likelihood of deformation exceeding freeboard all can involve calculations, as opposed to simpler models where the only values with distributions are event probabilities. The more complex procedure may be of great value when the breach probability is very sensitive to small changes in physical quantities, like the reservoir elevation at the time of the earthquake, the amount of settlement, or the residual undrained shear strength.

Exercise

Using the event tree in Figure 27-1 as a guide, develop an event tree to assess the risk probability of failure for the failure mode of deformation leading to transverse cracking and internal erosion through the cracks.

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